Punching strength of R.C. Flat slabs with moment transfer

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ABSTRACT
The paper presents results from an experimental and analytical study of interior columns subjected to punching in the presence of an unbalanced moment. The moment, characterized in the study by an eccentricity of the load in the column, is shown to have a significant effect in decreasing the punching strength. Punching shear reinforcement, while improving both the ultimate load and the deformability of the slab, is exhibiting a similar behavior with regard to the interaction of punching and unbalanced moment. Results from a parametric study show that, in most cases, unbalanced moments account for only a small decrease of the ultimate load capacity for interior columns, both in the presence of unsymmetrical loads and spans as in the case of imposed displacements.

Key words: Punching shear, brittle behavior, ultimate load, shear reinforcement, moment transfer, eccentric loading, laboratory tests, analytical studies.

1. INTRODUCTION

Unbalanced moments commonly occur in buildings with flat slabs, caused for instance by unequal spans or loading on either side of the column. Differences of temperature or differential creep between two adjacent floors results in differential displacements of the top and bottom of the columns, which induce moments in the slab-column connection, even if the columns, as is assumed for this study, do not participate in the horizontal load resisting system. In the presence of such moments, the phenomenon of punching becomes unsymmetrical, and the punching strength of the slab decreases. This phenomenon has been described by researchers [1,2,3,4] and is accounted for in current codes, with a varying level of detail [5,6,7,8].

The purpose of the present research project was to observe and quantify the effect of a moment on the punching load and to quantify the expected moments for typical configurations of internal columns in buildings. The tests are relatively large scale compared to previously tested configurations, the intent being to simulate as closely as possible typical building slabs in a scale around one half to two thirds of the prototype. The effectiveness of punching shear reinforcement in the presence of moment transfer was also investigated.

2. EXPERIMENTAL TESTING

A total of seven slabs of dimensions 3.0 x 3.0 x 0.15 m with a square column 0.30 x 0.30 m were tested in the testing setup shown in figure 1. The slabs were cast with normal strength concrete ($f_c \cong 35$ MPa, details in table 1). The cast were made in the normal configuration, after which the slabs were turned around to be tested in the inverse position as shown in figure 1. The load was introduced vertically downward, with the eccentrically located hydraulic ram introducing simultaneously the vertical force and the moment in the column, both remaining proportional. The slab is simply supported on knife edges fixed on steel beams so that the edges are free to lift. A total of seven slabs have been tested, with varying eccentricities of the loading: zero, approximately one-half and one time the column dimension (0, 0.16 and 0.32 m).

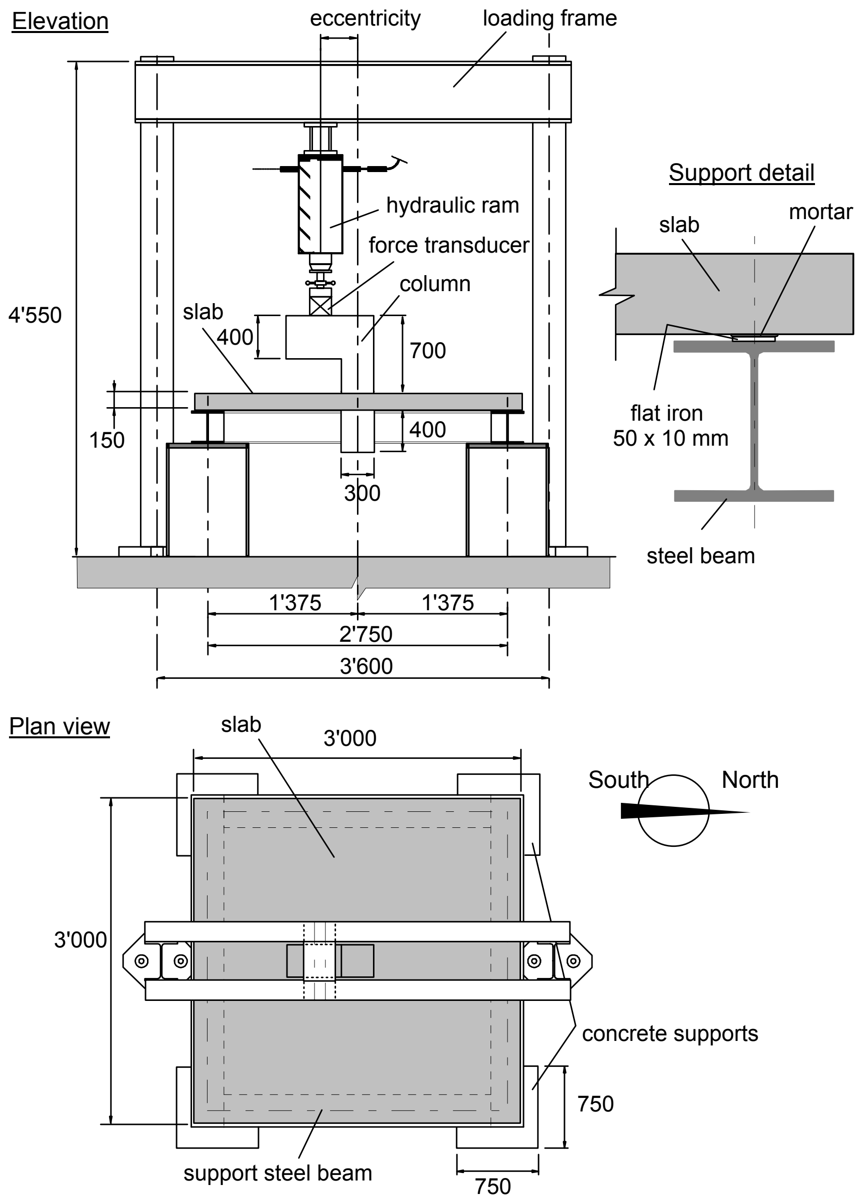
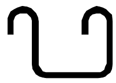


Fig. 1 Experimental test setup(dimensions in mm)

All slabs had similar reinforcement layouts, with reinforcement on the tension side only. The flexural reinforcement ratio was 1.0 % for slabs without shear reinforcement and 1.3 % for slabs with shear reinforcement (fig. 2). The nominal average effective depth is 0.121 m. The shear reinforcement used was either in the form of stirrups or of two-headed studs (table 1).

Table 1: Main parameters for all test specimens

Specimen	Eccentricity e [mm]	Longitudinal reinforcement	Punching reinforcement		Concrete properties at 28 days		
					f_{cm} [MPa]	f_{ctm} [MPa]	E_{cm} [GPa]
P0A	0	1.0 % ∅14 s = 120 mm	none		34.6	2.6	33.9
P16A	160				38.6	2.6	35.6
P30A	320				30.4	2.6	32.1
PP16A	160		48 stirrups ∅10 (96 legs)		37.7	2.5	31.4
PP0B	0	1.3 % ∅16 s = 120 mm			46.1	2.8	35.5
PP16B	160				39.5	2.4	32.4
Anco16B	160		84 studs ∅14		35.2	2.3	-

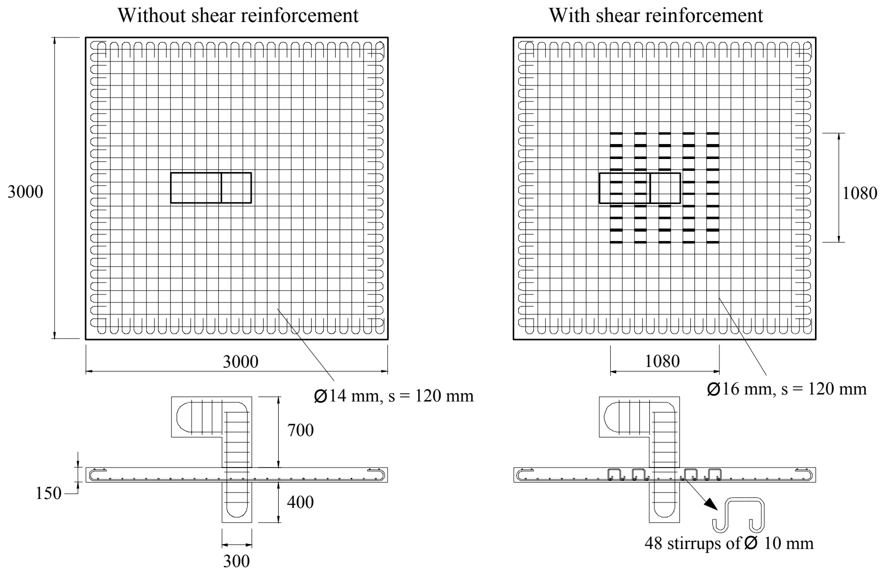


Fig. 2 Longitudinal and shear reinforcement (dimensions in mm)

The tests were performed with a deformation-controlled hydraulic ram with a constant loading rate of 4 kN per minute. During every test, the load was applied in steps of 40 kN. Between load steps the deformation was kept constant for 10-15 minutes for inspection and measurements. After the peak load was reached, the deformation was further increased to record the post-punching behavior of the slabs. The tests ended when the column had penetrated into the slab or when the rotation of the column exceeded 5 %.

The slabs were instrumented with:

- 35 inductive displacement sensors;
- 32 strain gauges (Omega gauges) on the compression side in radial and tangential directions;
- 4 strain gauges glued on the longitudinal bars near the column in the punching zone;
- 3 inclinometers on the column;
- 2 inductive displacement sensors measuring the opening of the punching cracks across the thickness of the slab;
- 1 force sensor on the jack;
- 66 manual measurements (Demec extension gauges) on the tension side to measure opening of the cracks.

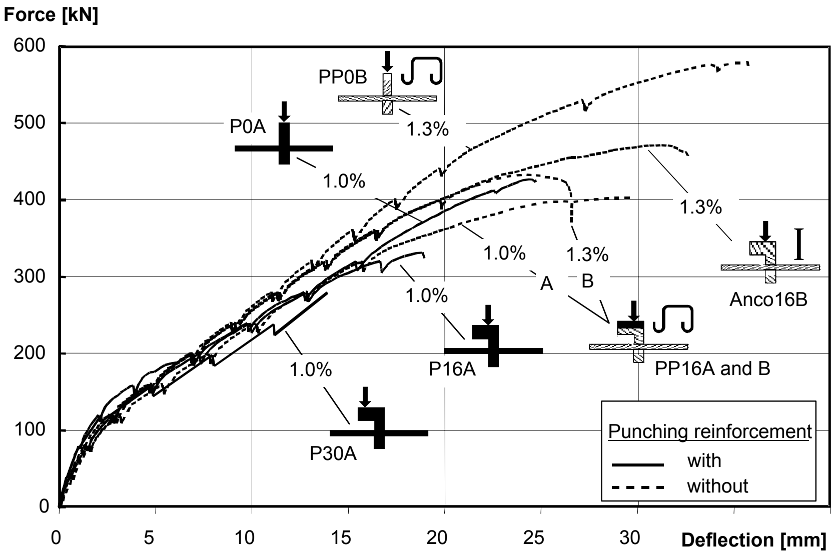


Figure 3: Load-deflection curve at mid-span of the slabs for all 7 tests

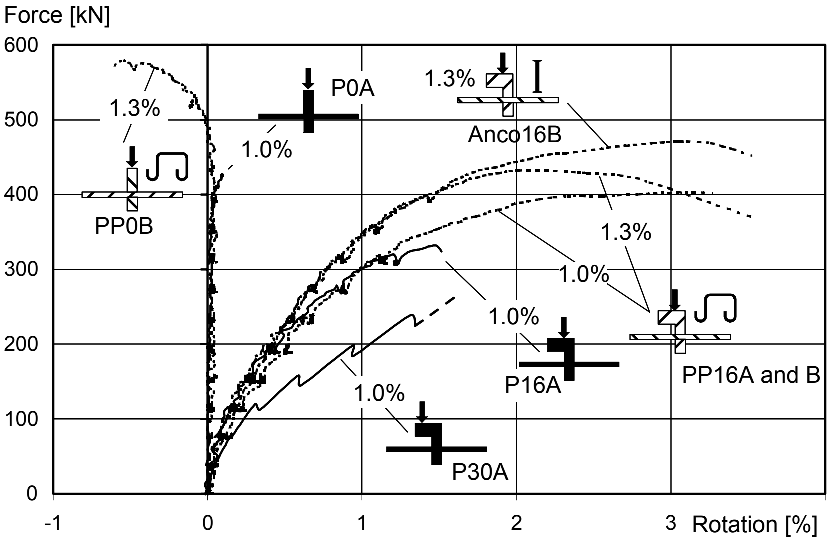


Figure 4: Load – column rotation curve for all test specimens

Table 2: Ultimate load for all test specimens

Specimen	*e* [m]	max. deflection [mm]	Ultimate load V_u [kN]	Decrease compared to case w/o eccentricity
P0A	0	25	423	-
P16A	0.16	19	332	22 %
P30A	0.32	14	270	36 %
PP0B	0	36	579	-
PP16A	0.16	30	403	30 %
PP16B	0.16	27	432	25 %
Anco16B	0.16	33	470	20 %

Measurements from electrical gages and displacement sensors were recorded every minute on a computer. The crack pattern was inspected and the manual measurement of radial and tangential deformation at the bottom surface of the slab (tension) were performed at the end of each load step.

Figure 3 shows the maximum slab defection versus the force applied by the hydraulic ram. The initial slope is almost identical for all slabs because it depends only on the shape and the modulus of elasticity of the slab. For all slabs the first crack appears approximately at the same load level independently from the eccentricity (between the second and the third load step at approximately 100 kN). After that first cracks develops, the slope of curves is governed by the longitudinal reinforcement. The slope for the three slabs with shear reinforcement ($\rho_{long} = 1.3$ %) is higher than without shear reinforcement ($\rho_{long} = 1.0$ %). The larger displacement at failure of slabs with shear reinforcement is evident, and this also the case in the presence of an eccentricity of the load.

This is further illustrated in figure 4 which shows the maximum column rotation as a function of the applied load.

Table 2 summarizes the maximal load achieved by all test specimens and shows the decrease in punching strength compared to the corresponding case without eccentricity. Figure 5 shows these values in a diagram that highlights reduction of ultimate punching strength as a function of the eccentricity.

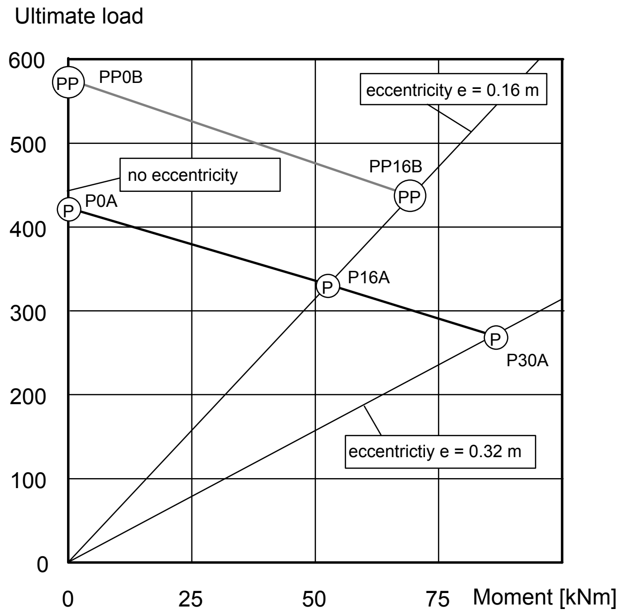


Figure 5: Decrease in punching strength as a function of the eccentricity of the load

The prediction corresponds rather well to values found by various code methods, although the absolute value of the ultimate load varies considerably from one code to the other. The rate of reduction of the strength in the presence of an eccentricity is approximately the same for slabs with punching reinforcement than for slabs without punching reinforcement. Further details on the testing procedure and the interpretation of the results can be found in [9,10,11].

3. Parametric study on the effect of unbalanced moments in buildings

A parametric study was performed to assess the magnitude of unbalanced moments to be expected in typical office building situations. A wide range of parameters was included in the study, that should cover most common cases. In special cases where the loads are significantly larger than usual, or if the geometry exceeds the limits of the study, it is strongly recommended that a particular attention be given in assessing the level of unbalanced moments in the slab-column connection. The parametric study was performed using a linear elastic shell program. The typical configuration used in the calculations is shown in figure 6. The main parameters of the study are a variable load in adjacent spans, variable spans, the size of the square column and the depth of the slab. Table 3 summarizes the geometrical parameters used in the study, and figure 7 shows the load combinations used for the calculations. The loads are representative of typical office buildings, the load combinations and load factors are those of the current Swiss standard SIA 160 [12].

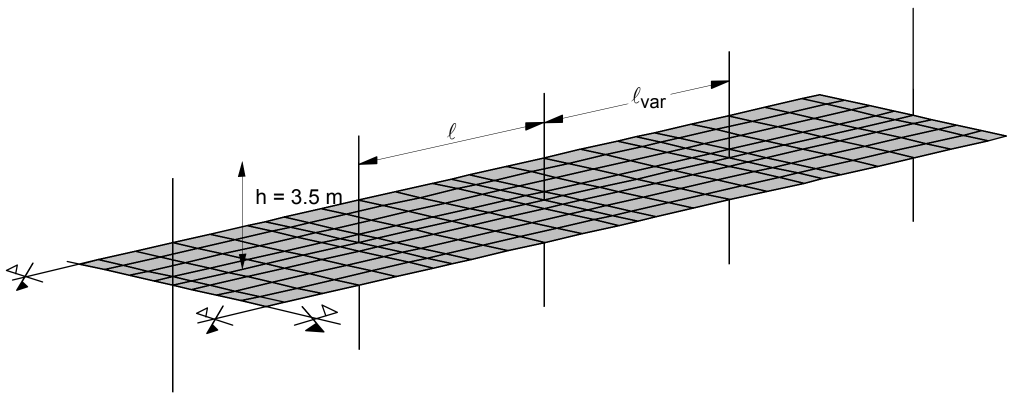


Figure 6: Finite Element model for the parametric study

Table 3: Geometric data for the parametric study

Span ℓ [m]	6, 8 and 10 m
$\Gamma = \ell_{var} / \ell$	1, ¾, ½ and ¼
Constant slab thickness [m]	0.20, 0.25, 0.30, 0.50 and 1.0 m
Square column side dimension c [m]	0.20, 0.30, 0.40, 0.50, 0.70 and 1.0 m

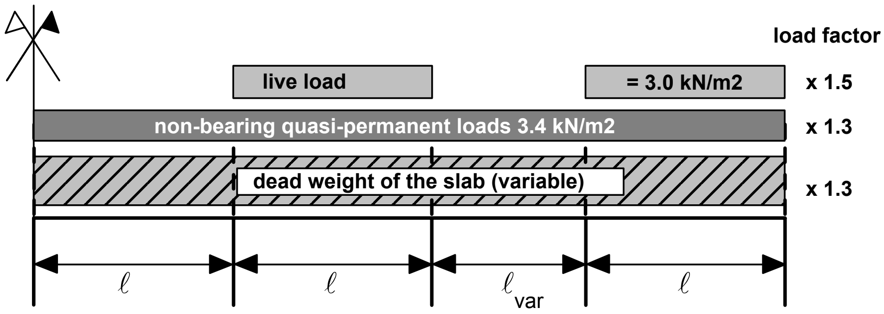


Figure 7: Model for vertical loads for the parametric study. The location of the live load induces maximal moments in interior columns

The moments determined from the numerical simulations were used to determine the critical shear using the approach of the CEB-FIP MC 90 [6], equ. 1. The results are presented using the ratio $\alpha = \tau_M / \tau_V$ defined in equation 2, which characterizes the increase in shearing stress caused by the unbalanced moment. This value is related to the β value given in EC2 to increase the column load to account for unbalanced moments : $\beta = 1 + \alpha$. In EC2, β = 1.15 for interior columns, so α_{max} would be around 0.15.

$$\tau_{sd} = \tau_V + \tau_M = \frac{V}{u \cdot d} + \frac{KM_{nb}}{W \cdot d} \qquad (1)$$

$$\alpha = \tau_M / \tau_V \qquad (2)$$

As figure 8 shows, the value of α is strongly dependant on the relative dimensions of the slab and the column. For typical cases, however, the increase in shearing stress is less than 20 %. In fact, focusing on dimensions close to reality, one can see in figure 9 that, in the case of constant spans and variable loads, the maximum increase for building slabs is always less than 15 %, thus confirming the values given by the Eurocode. The presence of variable spans increases slightly these values.

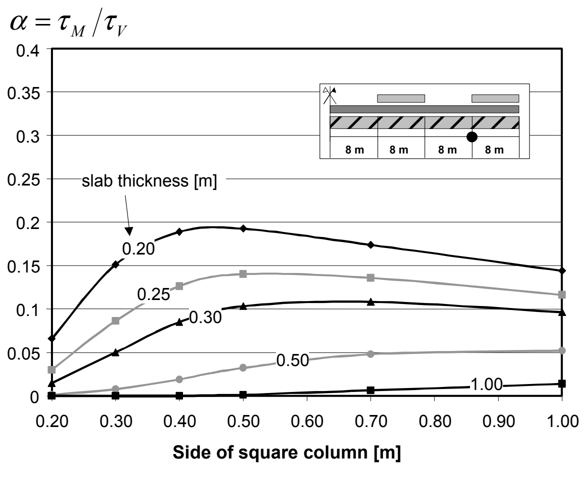


Figure 8: Increase in shearing stress α for the first interior connection for a slab with constant span of 8 m

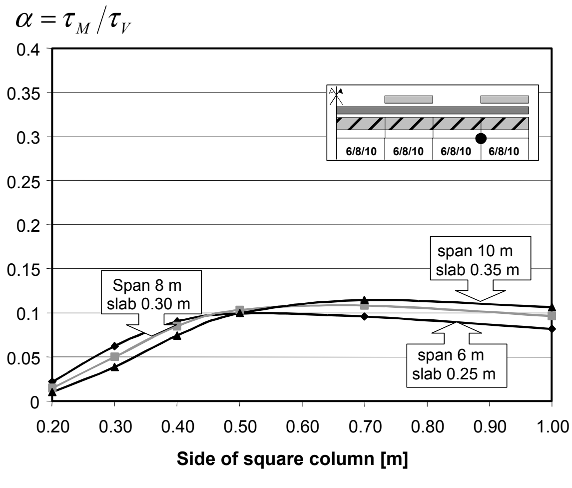
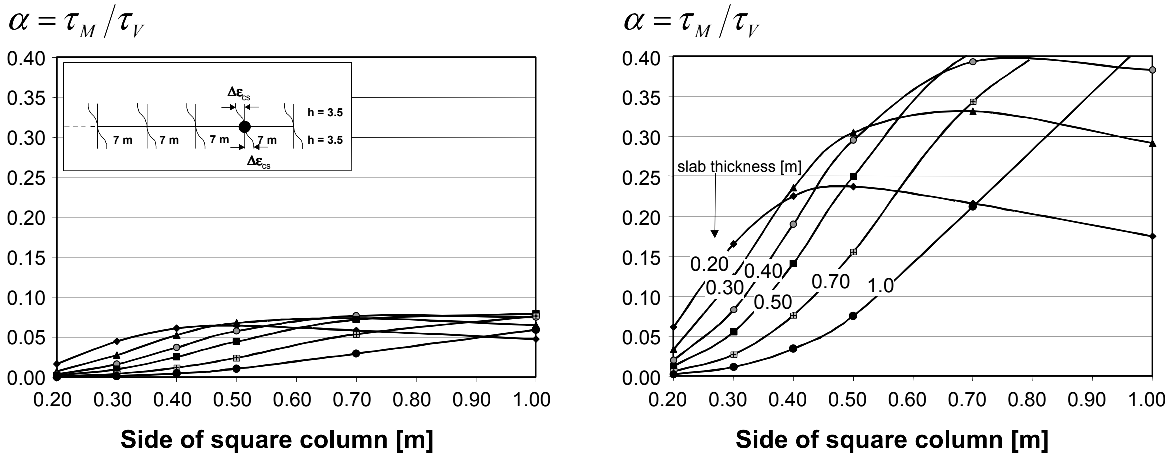


Figure 9: Increase in shearing stress α induced by unbalanced moment for three representative slabs with equal spans

The effect of relative imposed deformations between adjacent floors, induced by shrinkage and temperature, was also investigated. This effect is strongly dependant on the distance from the fixed point (concrete core) to the farthest column. In small buildings and in buildings with numerous expansion joints, this effect is negligible. In large buildings with fewer expansion joints, however, these effects become significant. If the construction sequence allows for only a short period (typically one month) between the casting of floors, figure 10a shows that the increase in shearing stress is small, even for a distance to the fixed point of 50 m. If the construction sequence is such that a whole year elapses between the construction of floors, figure 10b shows that a tremendous increase in the shearing stress is to be expected from differential shrinkage. In such a case, temporary shrinkage joints need to be provided. Further details about the parametric study can be found in [9].



a) 28 days between casting of floors

b) One year between casting of floors

Figure 10: Increase in shearing stress α for two casting sequences of floors. Conditions : ℓ = 7 m, ℓ_{pf} = 50 m, E^* = 11 kN/mm^2, plastic concrete, RH= 50 %

To summarize the main results from the parametric study, table 4 was prepared to enhance the β-value indicated by Eurocode for interior columns. It has been observed that only columns located in the immediate vicinity of the first field, or of a discontinuity in spans are susceptible to a substantial increase in shearing stresses. Thus, the values in table 4 generally apply to the first interior connection. All other interior columns can be checked using $\beta = 1.0$.

Table 4: Recommended β-values for interior columns in non-sway buildings. These values do not account for horizontal displacements.

<table>
<tr><td></td><td></td><td colspan="3">Side of square column [m]</td></tr>
<tr><td></td><td></td><td>$c = 0.20$</td><td>$0.20 < c \leq 0.30$</td><td>$c > 0.30$</td></tr>
<tr><td rowspan="3">Ratio Γ of spans of either side of an interior column</td><td>Γ = 1</td><td colspan="2">1.05* (1st interior connection)</td><td>1.10*
(1st interior connection)</td></tr>
<tr><td>Γ ≥ ¾</td><td>1.05</td><td>1.10</td><td>1.15</td></tr>
<tr><td>Γ < ¾</td><td>1.05</td><td>1.15</td><td>**Calculation of unbalanced moment M_{nb} required**</td></tr>
</table>

** for ordinary interior columns, $\beta = 1.0$*

4. CONCLUSIONS

The seven punching tests of this research project have made an interesting contribution to the understanding of punching in the presence of unbalanced moments, The observed decrease of the punching strength can be in excess of 30 % for large eccentricities of the column load. Punching reinforcement has been shown to significantly increase the ductility of the punching phenomenon, allowing much larger rotations of the column at failure. The decrease in capacity induced by a moment is similar for columns with and without shear reinforcement.

The parametric study performed on a large number of slabs representative of typical office buildings has shown that the actual eccentricity is usually small, resulting in an increase of the nominal shearing stress of less than 15 % in most cases. A set of more precise values have been proposed to improve on the values of the β-factor presently used in Eurocode 2.

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